
भू-स्खलन का स्थल विशिष्ट जाँच और
स्थिरता विश्लेषण — दिशानिर्देश

Site Specific Investigation and
Stability Analysis of Landslides —
Guidelines

ICS 93.020

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FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Hill Area Development Engineering Sectional Committee, had been approved by the Civil Engineering Division Council.

Landslides of different types occur at specific locations under certain topographic and geologic conditions. Therefore, it is important to carry out detailed investigations to study the failure mechanism of landslides to conduct slope stability analyses, to design a scheme of instrumentation for their deformation monitoring and finally to design appropriate control measures.

Prior to any site specific investigation of landslides, it is important to collect the existing information pertaining to the particular landslide in the form of literature, maps, history and data. This prior knowledge, if exists, will form a basis for drafting a detailed investigation plan to be carried out for the landslide.

This standard has, therefore, been formulated to provide general guidelines for site specific investigations and stability analysis of existing and potential landslide slopes in hilly terrain, and consequent large scale mapping of hill slope.

The composition of the Committee responsible for the formulation of this standard is given in Annex B.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 1960 'Rule for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be same as that of the specified value in this standard.

Indian Standard

SITE SPECIFIC INVESTIGATION AND STABILITY ANALYSIS OF LANDSLIDES — GUIDELINES

1 SCOPE

This standard covers general guidelines for site specific investigations and stability analysis of existing and potential landslide slopes in hilly terrain. In this context, site specific study refers to the large scale mapping and investigations of a hill slope.

2 REFERENCES

The standards given in Annex A contain provisions which through reference in this text constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards.

3 SITE SPECIFIC LANDSLIDE INVESTIGATION

With a preliminary knowledge on the landslide based on the existing information and a reconnaissance survey of the landslide area, a site specific detailed investigation plan can be developed and executed to delineate the aerial extent and a general direction of movement of the landslide zone, to assess the geology and geologic structure and to decipher the cause of sliding. The investigation should not include just the actual landslide area, but also include surrounding peripheral areas of the landslide boundary.

The site specific detailed landslide investigation encompasses surface, sub-surface and laboratory investigations as described in 3.1 to 3.3.

3.1 Surface Investigation

The surface investigation of a landslide includes topographic and geological investigations based on field survey and collection of samples for geotechnical characterization through laboratory testing. Also, a high resolution remote sensing data (up to 5 m spatial resolution) may be used for facilitating the surface investigation and may be useful in delineating the landslide features with a synoptic view.

3.1.1 Topographic Investigation

The topographic survey of the landslide should be carried out on 1 : 500 to 1 : 1 000 scale using total station

survey instrument to prepare the elevation contour map with 1 m contour interval. The contouring should not only be done for the actual landslide body, but also include the surrounding area beyond the landslide boundary on uphill side at least up to 50 m or up to the ridge top of the hill slope and on downhill side up to the valley. The topographic/contour map will serve as the base map for all the investigations.

The boundary of the landslide from crown to toe along with all the landslide features, such as the main scarp, minor scarps, transverse/longitudinal/radial cracks observed, slope breaks, zone of initiation, zone of transportation and zone of accumulation, etc, should be mapped on the contour map. The surface drainages/streams should be mapped on the contour map. The slope angle, slope aspect, break in slope and slope morphology (convex/concave/straight) shall be assessed from the contour map and field survey.

At least three profile sections along the central portion, left flank and right flank of the landslide area shall be drawn using the contour map. The profile sections should be located so as to provide information that will be needed for planning sub-surface investigations and stability analyses. All the topographic features of the landslide should be marked on the profile sections. These profile sections should be augmented with geological and other information from time to time with the progress of investigation.

3.1.2 Geological Investigation

Local geologic and tectonic information provides a useful basis for an understanding of the general nature of the site geology. The rock types, surficial deposits and structural features of the landslide area should be mapped through field survey and portrayed on the base contour map. The geologic map should provide reasonably detailed information on surficial deposits, including landslide deposits. The depth of the surficial deposits may be estimated from the cut slopes on road side of the landslide site. The structural discontinuities, such as bedding/foliation planes and joints are to be mapped in detail in the field for *in-situ* rock exposures at the landslide site. The dip slope relationship of the landslide area should be established with the stereo plots of slope, bedding/foliation and other structural discontinuities data. This will help in assessing the mode of failure of rock slides. Rock slopes should be

classified based on Slope Mass Rating (SMR) as per IS 13365 (Part 3). Other important aspects of the site to document include the distribution and thickness of colluviums and depth of bedrock weathering.

Based on the collected information of the landslide area, an engineering geological map of the landslide should be prepared on the base contour map. The engineering geological map of the landslide area should include all the landslide features, surface drainages/streams, surficial deposits, bedrock exposures and structural discontinuities. The purpose of the on-site engineering geologic mapping is to document surface conditions, which, in turn, provides a basis for projecting subsurface conditions that may be relevant to the stability of the site.

The profile sections of the landslide should be updated with the surface geological information and interpretations. Based on the detailed engineering geological mapping, the mode of failure of the landslide slope, such as circular mode of debris slide, translational mode of debris slide on the bed rock, planar/wedge/toppling failure modes of rock slide should be assessed.

Hence, an interpretive result of engineering geological mapping will be the classification of slopes in the landslide area in terms of stability.

3.1.3 Collection of Samples

Soil and rock samples from the surface of the landslide area should be collected judiciously. The samples may be collected from different contour levels along or in the near vicinity of the profile sections selected for stability analyses. These samples are taken primarily for laboratory testing for the measurement of *in-situ* material properties. Samples of the weathered and/or colluvial soil, the existing landslide rupture materials and the weakest components of the *in-situ* rock units should be taken for laboratory measurement of engineering properties. Special care should be taken to obtain oriented samples of existing and exposed zones of weakness or rupture surface. It is the responsibility of the field supervising geologist or engineer to accurately label and locate the collected samples.

Soil samples that may be obtained from surface or from test pits belong to one of the two basic categories such as disturbed and undisturbed samples. Disturbed samples are collected primarily for soil classification tests where the preservation of the soil structure is not essential or for remoulding in the laboratory and subsequent strength tests. Undisturbed samples do not entirely represent truly undisturbed soil or rock conditions because the process of sampling and transporting inevitably introduces some disturbance into the soil or rock structure.

3.2 Sub-surface Investigation

Sub-surface investigation by the engineering geologist and civil engineer should be conducted at locations

considered most likely to reveal any sub-surface conditions which may indicate the potential for landslide failures. Such investigation should locate and define geometry of the bedding/foliation and fracture surfaces, geological contacts, faults and other discontinuities as well as actual landslide rupture surfaces.

Sub-surface investigation/exploration methods can be classified as direct and indirect methods. Direct methods, such as test borings/drilling and the excavation of test pits or trenches, allow the examination of the earth materials, usually with the collection of samples from different depths. Indirect method, such as geophysical surveys, provides an assessment of material type across depth. It would be recommended to follow direct investigations after reviewing results of indirect investigations.

3.2.1 Geophysical Investigation

Geophysical survey techniques, such as seismic and electrical surveys of the landslide area may provide an alternative method for sub-surface investigation to understand the approximate conditions of the landslide and surrounding area. These techniques measure the vertical and lateral variation of physical properties of the ground. They can be particularly useful in areas where direct methods, such as drilling and test pits are restricted. Geophysical methods are also generally used to support more traditional techniques (that is drilling and test pits) and should be correlated with the interpretation of intrusive investigation to improve confidence in results. Standard data collection techniques for geophysical survey should be used.

3.2.2 Investigation through Drilling

The locations and depths of bore holes for sub-surface investigation should be judiciously decided based on the interpretation of surface engineering geological investigations and geophysical scan line survey to reveal any sub-surface condition related to landslide failures. The drilling locations may preferably be along the profile sections or in the near vicinity of the profile sections. The depth to which drilling should extend can be difficult to define in advance. However, profile cross-sections from the surface engineering geological investigation can be helpful in planning the depths of bore holes required in investigation. In general, drilling should extend deep enough to identify materials that have not been subjected to movements in the past but might be involved in future movements and to clearly identify and extend beyond the underlying stable materials or bedrocks.

The drill holes should be properly logged (as per IS 4464) to extract as many information as possible. The bore hole logs should include the detailed information as: geologic and soil description, colour, hardness, lithologic description, degree of weathering, alterations and fractures, strike and dip of bedding/foliation and other joints/discontinuities, core recovery, RQD (Rock

Quality Designation) and ground water levels. In addition, during drilling the information on drilling rate, water loss, return water colour and collection of water washed samples should be undertaken.

Using the drilling logs, the surface of rupture or slide plane should be assessed. Accordingly, the engineering profile sections of the landslide area should be improved and updated with the incorporation of sub-surface data

3.2.3 Collection of Samples

During the intrusive technique of sub-surface investigation through bore holes, all the materials (soil/debris/weathered rock/fresh rock) extracted from the sub-surface should be preserved in core boxes with proper label along the depths. These core boxes may be transported to the geotechnical laboratory for characterization of different materials across the depths of the landslide area.

3.3 Laboratory and Field Testing

In order to conduct slope stability analyses and to design appropriate control measures for landslides, physical as well as engineering properties of the soil and rock materials should be determined through field and laboratory investigations.

3.3.1 For characterizing the debris and soil the following tests should be done as per the Indian standards given below:

- a) Classification and identification of soils as per IS 1498.
- b) Penetration test for soils as per IS 2131.
- c) Thin walled tube sampling of soils as per IS 2132.
- d) Preparation of dry soil samples for various tests as per IS 2720 (Part 1).
- e) Determination of water content as per IS 2720 (Part 2).
- f) Determination of specific gravity as per IS 2720 (Part 3).
- g) Determination of unconfined compressive strength as per IS 2720 (Part 10).
- h) Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure as per IS 2720 (Part 12).
- j) Direct shear test for soils containing gravel as per IS 2720 (Part 39/Sec 1).
- k) *In-situ* vane shear test for soils as per IS 4434.

3.3.2 For characterizing the rock mass the following tests should be done as per the Indian standards given below:

- a) Uniaxial jacking test for deformation modulus of rock as per IS 7317.

- b) *In-situ* shear test on rock as per IS 7746.
- c) Quantitative description of discontinuities in rock mass for Drill core study as per IS 11315 (Part 12).
- d) Hardness of rock as per IS 12608.
- e) Direct shear strength of rock joints as per IS 12634.
- f) *In-situ* determination of rock mass deformability using a flexible dilatometer for Radial displacement as per IS 12955 (Part 2).
- g) Predicting of engineering properties (RMR) as per IS 13365 (Part 1).
- h) Determination of slope mass rating (SMR) as per IS 13365 (Part 3).
- j) Geological Strength Index (GSI) as per IS 13365 (Part 4).

3.3.3 For characterizing of the intact rock specimens or on lumped rock samples following tests should be done as per the Indian standards given below:

- a) Point load strength index of rocks as per IS 8764.
- b) Unconfined compressive strength of rock materials as per IS 9143.
- c) Preparation of rock specimen for laboratory testing as per IS 9179
- d) Determination of modulus of elasticity and Poisson's ratio of rock materials in uniaxial compression as per IS 9221.
- e) Determination of slake durability index of rocks as per IS 10050.
- f) Determination of tensile strength by indirect tests on rock specimens as per IS 10082.
- g) Determination of water content, porosity, density and related properties of rock material as per IS 13030.
- h) Determination of strength of rock materials in triaxial compression as per IS 13047.

4 LANDSLIDE STABILITY ANALYSIS

The evaluation of stability of the hill slopes becomes very essential for the safe design, especially when the slopes are situated close to residential areas or when infrastructures are built on these slopes. A hill slope can fail due to one or the combination of the four mechanisms like circular sliding, plane sliding, wedge sliding and toppling. Circular failure occurs in rock mass if it can be justified as homogeneous medium. When the instability is dictated by the presence of pre-existing geological features like joints, bedding planes or faults then the failure will be in the form of plane sliding, wedge sliding or toppling (see Table 1 of IS 14680). To assess the stability of a hill slope in jointed rock masses, all these possible failure modes shall be checked.

A vast range of methods for the stability analysis of rock and mixed rock-soil slopes have been developed. These methods range from simple infinite slope and planar failure limit equilibrium techniques to sophisticated coupled finite/distinct element codes. Collection of field data and then implementation of the same is the portion where uttermost care should be taken. In the section below kinematic analysis is discussed in very brief followed by some detailed discussion to arrive at different input parameters required for hill slope stability analysis. Two different modelling techniques are discussed. In one case, the slope is represented as a continuum in which the effect of discontinuities may be considered by reducing the properties and strength of intact rock to those of rock mass. In another case, a combined continuum-interface numerical method may be adopted where the discontinuities are represented as interface elements in between the rock walls. In each case of numerical modelling of landslide slope, both static and dynamic (including earthquake parameters) analysis as well as analysis under dry and water saturated conditions shall be carried out. The loading conditions on the landslide slope such as dead loads (due to buildings and other infrastructures) and imposed loads (due to traffic, etc) shall be considered during numerical modelling.

4.1 Kinematic Method

Stereographic projection are used for the visualization and determination of the kinematic feasibility of rock slopes using friction cones, daylight and toppling envelopes, in addition to graphical and statistical analysis of the discontinuity properties. It is essential to note that such approaches recognize potential sliding failures involving single discontinuities or discontinuity intersections.

The planar wedge slide occurs under gravity alone where rock mass rests on an inclined geological discontinuity, such as bedding/weakness plane that day-light into the free slope. A caution should be exercised that a rock slope may fail by toppling if dip of cross joint is more than $90^\circ - \psi_f + \phi_j$, where ψ_f is the slope angle and ϕ_j is the sliding angle of friction along joint plane.

The failure of rock wedge (three-dimensional) occurs when two discontinuities strike obliquely across the slope face and their line of intersection daylights in the slope face. The rock wedge resting on these discontinuities may slide down along the line of intersection, provided that the inclination of this line is significantly greater than the sliding angle of friction. Alternatively, the wedge may slide on any plane of discontinuity. Further, the wedge may fail by floating due to high seepage pressures on the joints.

Discontinuity data for different joint sets are used to assess the factor of safety against sliding. All limiting equilibrium techniques share a common approach based on a comparison of resisting forces/moments

mobilized and the disturbing forces/moments. Methods vary, however, in the assumptions adopted in order to achieve a determinate solution.

4.2 Numerical Method

It is very crucial to choose the proper numerical modelling for rock slope stability analysis. Continuum modelling is best suited for the analysis of slopes that are comprised of weak rocks, and soil-like or heavily fractured rock masses. Limit equilibrium method, limit analysis method commercially available numerical code are useful in characterizing rock slope failure mechanisms. It is the responsibility of the engineer to verify whether they are representative of the rock mass under consideration. In case of a rock slope comprising multiple joint sets, which control the mechanism of failure, a discontinuum modelling approach may be considered as more appropriate.

4.2.1 Continuum Analysis

Rock slopes having weak rocks and soil-like or heavily fractured rock masses may be modelled as a continuum using a two dimensional, plane strain approach. The slope is represented as continuum in which the effect of discontinuities may be considered by reducing the properties and strength of intact rock to those of rock mass. Material properties are the most important input parameters for any numerical analysis. Hence, uttermost care has been taken to decide over the material properties. Most of the numerical codes consider the strength of rock mass as linear and generally require Mohr-Coulomb parameters, c (cohesion) and ϕ , angle of internal friction, of slope material as strength parameters.

Laboratory tests on small specimens cannot predict the shear strength properties of rock masses, *in-situ* tests which provide direct information on shear strength properties and deformability is necessary. However, *in-situ* tests are expensive and time consuming.

Value of cohesion, friction and tensile strength may be obtained by fitting straight line over the generalised Hoek-Brown (GHB) failure envelope. The fitting process involves balancing the areas above and below the Mohr-Coulomb plot. Required input parameters are uniaxial compressive strength (UCS), geological strength index (GSI), Hoek-Brown constant (m_i), Disturbance factor (D) and Elastic modulus of intact rock (E_i). The shear strength parameters obtained from the above method is the peak shear strength of the rock mass.

The main motive behind this generalised Hoek-Brown (GHB) empirical failure criterion was to develop a dimensionless equation that could be scaled in relation to geological information,

$$\sigma_1 = \sigma_3 + \sigma_{ci} \left(m_b \frac{\sigma_3}{\sigma_{ci}} + s \right)^a \quad \dots (1)$$

$$m_b = m_b \exp\left(\frac{GSI-100}{28-14D}\right); s = \exp\left(\frac{GSI-100}{9-3D}\right); \quad \dots (2)$$

$$a = \frac{1}{2} + \frac{1}{2} + \left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right)$$

The constants m_b , s and a may be obtained from GSI. A parameter D is introduced to deal with blast damage.

For GSI, see IS 13365 (Part 4).

The value of equivalent cohesion and friction angle depends on the value of maximum minor principal stress ($\sigma_{3\max}$). $\sigma_{3\max}$ can be calculated from the following relation for the slopes:

$$\frac{\sigma_{3\max}}{\sigma_{c-\max}} = 0.72 \left(\frac{\sigma_{c-\max}}{\gamma H} \right)^{-0.91} \quad \dots (3)$$

The equivalent angle of friction and cohesion for the rock mass may be obtained by fitting a linear relationship to the curve generated by solving the generalised Hoek-Brown (GHB) criteria for a range of minor principal stress (σ_3) values defined by $\sigma_t < \sigma_3 < \sigma_{3\max}$, where σ_t is the tensile strength of the jointed rock mass. Zero or a very nominal value of tensile strength for the rock mass may be considered.

Strength reduction technique is generally implemented to determine the factor of safety using finite element method satisfying Mohr-Coulomb's yield criterion.

The factor of safety of a slope defined in the strength reduction technique is the factor by which the original shear strength parameter is divided in order to bring the slope to the verge of failure. For Mohr-Coulomb material model the factored shear strength parameters c_f and ϕ_f are calculated as:

$$c_f = \frac{c}{SRF} \quad \dots (4)$$

$$\phi_f = \tan^{-1} \left(\frac{\tan \phi}{SRF} \right) \quad \dots (5)$$

where SRF is the 'Strength Reduction Factor'.

To find the true factor of safety, a systematic search is to be performed that will just cause the slope to fail. The final value of SRF obtained in the process will be the Factor of Safety (FOS).

In order to obtain a FOS of high precision with less calculation and a proper distribution of plastic zones in the critical equilibrium state, the Poisson's ratio, ν should be adjusted according to the principle that $\sin \phi \geq (1-2\nu)$ always holds as reducing the strength parameters c and ϕ . In most of the finite element analysis, the value of ν is kept invariant; if the $\sin \phi \geq (1-2\nu)$ is not satisfied then plastic zones are always over estimated. This phenomenon leads to an underestimated FOS. In order to avoid the above spurious failure

mechanism, one can select a large enough Poisson's ratio, ν so that $\sin \phi \geq (1-2\nu)$ is always satisfied as the shear strength parameters are reduced. But one should be careful while choosing a high Poisson's ratio, if the value of the Poisson's ratio is closer to 0.5 as it is prone to leading to numerical ill-condition.

4.2.2 Combined Continuum-Interface Analysis

In case of a rock slope comprising of multiple joint sets, which control the mechanism of failure, a discontinuum approach may be considered as more appropriate.

Combined continuum-interface numerical method focuses on the explicit simulation of interfaces through joint elements. In this method of analysis, discontinuity is represented as joint element which provides relaxed connectivity between two adjacent rock walls in contact. The degree of proportionality between force and displacement in these joint element is captured through the normal, k_n and tangential, k_s stiffness coefficients.

If the rock between the joints is intact rock, then the intact rock properties should be used. Rock between the joint sets may be assumed to be intact rock if all the joint sets are incorporated in the numerical model. In this combined method of continuum-interface analysis input parameters for joints are the most vital. Joint normal stiffness, k_n may be calculated using the deformation modulus of intact rock material and rock mass.

Ratio of $\frac{k_n}{k_s}$ generally ranges between $\frac{1}{10}$ and $\frac{1}{20}$ for slope stability problems; where low normal stresses are generally encountered. Assuming the value of $\frac{k_n}{k_s}$, shear stiffness, k_s may be obtained.

Since, laboratory tests on small specimens cannot predict the deformability of rock masses, *in-situ* tests which provide direct information on deformability are necessary. Empirical relations may be used to relate the rock mass deformation modulus to the deformation modulus of intact rock using GSI.

Detailed field and laboratory investigation need to be carried out to determine the strength and deformation properties of the existing joints. The following information may be collected from field observations. Details of joint roughness, orientation, frequency, infilling, joint aperture, continuity, joint wall compressive strength, characteristics of bedding and foliations.

The spacing of discontinuities in the numerical model may be an average value of the minimum and maximum spacing of individual joint in the field. The spacing of joints should be chosen very carefully, because with the decrease in the spacing, the chance of numerical instability increases and the numerical models with very closely spaced joints are computationally very expensive. So a balance between the joint spacing and computation time should be achieved.

ANNEX A

(Clause 2)

LIST OF REFERRED INDIAN STANDARDS

<i>IS No.</i>	<i>Title</i>	<i>IS No.</i>	<i>Title</i>
1498 : 1970	Classification and identification of soils for general engineering purposes (<i>first revision</i>)	8764 : 1998	Method of determination of point load strength index of rocks (<i>first revision</i>)
2131 : 1981	Method of standard penetration test for soils (<i>first revision</i>)	9143 : 1979	Method for the determination of unconfined compressive strength of rock materials
2132 : 1986	Code of practice for thin walled tube sampling of soils (<i>second revision</i>)	9179 : 1979	Method for the preparation of rock specimen for laboratory testing
2720	Methods of test for soils	9221 : 1979	Method for the determination of modulus of elasticity and Poisson's ratio of rock materials in uniaxial compression
(Part 1) : 1983	Preparation of dry soil samples for various tests (<i>second revision</i>)	10050 : 1981	Method for determination of slake durability index of rocks
(Part 2) : 1973	Determination of water content (<i>second revision</i>)	10082 : 1981	Method of test for the determination of tensile strength by indirect tests on rock specimens
(Part 3) : 1980	Determination of specific gravity, Section 1 Fine grained soils (<i>first revision</i>)	11315 (Part 12) : 1992	Method for the quantitative description of discontinuities in rock mass: Part 12 Drill core study
(Part 4) : 1985	Section 2 Fine, medium and coarse grained soils (<i>first revision</i>)	12608 : 1989	Methods of test for hardness of rock
(Part 10) : 1991	Grain size analysis (<i>second revision</i>)	12634 : 1989	Method of determination for direct shear strength of rock joints
(Part 12) : 1981	Determination of unconfined compressive strength (<i>second revision</i>)	12955 (Part 2) : 1990	Code of practice for in-situ determination of rock mass deformability using a flexible dilatometer: Part 2 Radial displacement
(Part 39/Sec 1) : 1977	Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure (<i>first revision</i>)	13030 : 1991	Method of test for laboratory determination of water content, porosity, density and related properties of rock material
4434 : 1978	Direct shear test for soils containing gravel, Section 1 Laboratory test	13047 : 1991	Method for determination of strength of rock materials in triaxial compression
4464 : 1985	Code of practice for <i>in-situ</i> vane shear test for soils (<i>first revision</i>)	13365	Quantitative classification system of rock mass — Guidelines
7317 : 1993	Code of practice for presentation of drilling information and core description in foundation investigation	(Part 1) : 1998	RMR for predicting of engineering properties
7746 : 1991	Code of practice for uniaxial jacking test for deformation modulus of rock (<i>first revision</i>)	(Part 3) : 1997	Determination of slope mass rating (SMR)
	Code of practice for <i>in-situ</i> shear test on rock (<i>first revision</i>)	(Part 4) : 2014	Geological Strength Index (GSI)
		14680 : 1999	Landslide control — Guidelines

ANNEX B*(Foreword)***COMMITTEE COMPOSITION**

Hill Area Development Engineering Sectional Committee, CED 56

<i>Organization</i>	<i>Representative(s)</i>
In Personal Capacity, (H No-FFA, Block - G Priya Sudarsana Apts, Melur Road, Srirangam Tiruchirappalli, Tami Nadu)	DR R. ANBALAGAN (Chairman)
AIMIL Ltd, New Delhi	SHRI SHARIQUE KHAN SHRI AMAN KHULLAR (<i>Alternate</i>)
Border Roads Organization, New Delhi	BRIG A. K. DAS LT COL PARIKSHIT MEHRA (<i>Alternate</i>)
Building Materials & Technology Promotion Council, New Delhi	DR SHAILESH KUMAR AGARWAL
Central Public Works Department, Shillong	SHRI SHEO PRASAD SINGH SHRI PREM CHAND MATHUR (<i>Alternate</i>)
Central Soil & Materials Research Station, New Delhi	DR MANISH GUPTA SHRI HARI DAS (<i>Alternate</i>)
Central Water Commission, New Delhi	DIRECTOR HCD (NW&S) DIRECTOR (SARDAR SAROVAR) (<i>Alternate</i>)
CSIR - Central Building Research Institute, Roorkee	SHRI ASHOK KUMAR SHRI AJAI CHAURASIA (<i>Alternate</i>)
CSIR - Central Institute of Mining & Fuel Research, Roorkee	DR R. K. GOEL SHRI ANIL SWARUP (<i>Alternate</i>)
CSIR - Central Road Research Institute, New Delhi	DR KISHOR KUMAR DR PANKAJ GUPTA (<i>Alternate</i>)
Defence Terrain Research Laboratory/(DRDO), Delhi	DR L. K. SINHA
Department of Science & Technology, Ministry of Science & Technology, New Delhi	SHRI BHOOP SINGH
G. B. Pant Institute of Himalayan Environment Development, Almora	SHRI KIREET KUMAR SHRI SOUKHIN TARAFDAR (<i>Alternate</i>)
Geological Survey of India, Kolkata	SHRI M. S. BODAS DR T. B. GOSHAL (<i>Alternate</i>)
Himalayan Environmental Studies & Conservation Organization, Dehradun	DR ANIL JOSHI
Housing & Urban Development Corporation Ltd, New Delhi	CHAIRMAN & MANAGING DIRECTOR
Indian Army Corps of Engineers, YOL CANTT	BRIG VIRENDER SINGH KATTARYA
Indian Institute of Remote Sensing, Dehradun	DR P. K. CHAMPATI RAY DR AJANTA GOSWAMI (<i>Alternate</i>)
Indian Institute of Technology Roorkee, Roorkee	DR M. N. VILADKAR DR MAHENDRA SINGH (<i>Alternate</i>)
Indian Institute of Technology Bombay, Mumbai	REPRESENTATIVE
Konkan Railway Corporation, Navi Mumbai	SHRI RAJESH AGARWAL
Maccaferri Environmental Solutions Pvt Ltd, Gurugram	SHRIMATI MINIMOL KORULLA DR H. S. MANDAL (<i>Alternate</i>)

<i>Organization</i>	<i>Representative(s)</i>
Military Engineer Services, Engineer-in-Chief's Branch, Integrated HQ of Ministry of Defence, (Army), New Delhi	SHRIMATI RIVOO MAHENDRU SHRI S. K. MISHRA (<i>Alternate</i>)
National Centre for Seismology, New Delhi	DR O. P. MISHRA DR H. S. MANDAL (<i>Alternate</i>)
National Highway Authority of India, New Delhi	DR RAM KUMAR SHRI R. P. SINGH (<i>Alternate</i>)
National Hydroelectric Power Corporation Ltd, Hepali	SHRI S. MURUGAPPAN
National Institute of Disaster Management, New Delhi	REPRESENTATIVE
National Institute of Hydrology, Roorkee	SHRI R. D. SINGH DR SUDHIR KUMAR (<i>Alternate</i>)
Rail Vikas Nigam Limited, New Delhi	SHRI SUMIT JAIN
SJVN Limited, Shimla	SHRI A. K. CHADHA
Snow Avalanche Study and Establishment, Chandigarh	COL P. MATHUR
THDC India Limited, Rishikesh	SHRI T. S. ROUTELA SHRI AJAY KUMAR (<i>Alternate</i>)
Wadia Institute of Himalayan Geology, Dehradun	DR S. K. BARTARYA
BIS Directorate General	SHRI SANJAY PANT, SCIENTIST 'F' AND HEAD (CIVIL ENGINEERING) [REPRESENTING DIRECTOR GENERAL (<i>Ex-officio</i>)]

Member Secretary

DR MANOJ KUMAR RAJAK
SCIENTIST 'D' (CIVIL ENGINEERING), BIS

Bureau of Indian Standards

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This Indian Standard has been developed from Doc No.: CED 56 (12161).

Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

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Published by BIS, New Delhi